MECHANICAL PROPERTIES OF SOFT CLAY STABILIZED WITH CEMENT-RICE HUSKS (RH)

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A thesis submitted in fulfillment of the requirement for the award of the Degree of Master of Engineering (Civil-Geotechnics)

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SEPTEMBER 2011

ABSTRACT

Chemical stabilization has been extensively used for the improvement of soft clay soils, in enhancing the shear strength and limiting the deformation behaviours. Cement is widely used as a stabilizing material for soils, but the increasing price is causing economic concerns among practitioners and clients alike. The quest for alternative cheaper stabilizing agents is therefore more urgent than before. Rice husk is a major agricultural waste in Malaysia and the common disposal method of open burning has notoriously contributed to environmental pollution. The possibility of admixing rice husks with cement for stabilizing soft soils could be a solution to both problems. This study was aimed at assessing the usefulness of cement-rice husks as an effective soil stabilizer for improving the mechanical properties of clay soils. Laboratory experiments were carried out on a stabilized soft clay to study the inter-relationships between shear wave velocity, one-dimensional compressibility and unconfined compressive strength. Bulk clay samples were collected from the Research Centre for Soft Soils (RECESS) of UTHM. The stabilized specimens were prepared with the clay admixed with 5 % and 10 % cement and various quantities of rice husks, then compacted into cylindrical specimens measuring 38 mm in diameter and 76 mm high. The specimens were then left to cure for different periods up to a month. The stabilized specimens were observed to undergo increase in stiffness and strength, as well as significant reduction in compressibility, highlighting the great potential of cement-rice husk as an alternative soft soil stabilizer.

Keywords: Clay soil stabilization, cement, rice husk, shear wave velocity, one-dimensional compressibility, unconfined compressive strength

ABSTRAK

Penstabilan secara kimia telah digunakan secara meluas dalam penstabilan tanah, untuk menaikkan nilai kekuatan ricih tanah dan menghadkan ciri-ciri pemendapan tanah. Simen telah lama digunakan sebagai bahan penstabil tanah, tetapi kenaikan harganya merisaukan ekonomi pengguna yang terbabit. Oleh itu, permintaan terhadap agen penstabil yang lebih murah sangatlah diutamakan. Sekam padi merupakan antara bahan buangan utama daripada sektor pertanian di Malaysia dan kaedah pelupusannya iaitu secara pembakaran terbuka telah menyumbang kepada berlakunya pencemaran udara. Kecenderungan untuk mencampurkan sekam padi dengan simen dalam penstabilan tanah lembut mungkin merupakan jalan penyelesaian terhadap kedua-dua masalah tersebut. Matlamat kajian ini adalah untuk mengkaji kegunaan simen-sekam padi sebagai bahan penstabil tanah yang berkesan dalam memperbaiki ciri-ciri mekanikal tanah liat. Ujikaji-ujikaji makmal dijalankan terhadap tanah liat terstabil untuk mengkaji hubungan antara halaju gelombang ricih, pemampatan satu dimensi dan kekuatan mampatan tak terkurung. Sampel tanah liat diambil dari Pusat Penyelidikan Tanah Lembut (RECESS), UTHM. Spesimen yang terstabil disediakan dengan mencampurkan tanah liat dengan 5 % dan 10 % simen dan pelbagai kuantiti sekam padi. Kemudiannya, dimampatkan menjadi spesimen silinder berukuran 38 mm dan ketinggian 76 mm. Spesimen-spesimen tersebut kemudiannya diawet pada tempoh berlainan sehingga sebulan. Spesimen-spesimen tersebut dipantau bagi memastikan terdapatnya kenaikan nilai-nilai kekukuhan dan kekuatannya, serta penurunan mendapan yang efektif, yang menunjukkan terdapatnya potensi untuk simen-sekam padi sebagai bahan penstabil alternatif untuk tanah liat.

Kata kunci: *penstabilan tanah liat, simen, sekam padi, halaju gelombang ricih, pemampatan satu dimensi dan kekuatan mampatan tak terkurung.*

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LIST OF SYMBOLS AND ABBREVIATIONS

(τ)	-	time shift
A	-	cross section area
Al_2O_3	-	Alumina
BE	-	Bender element
C	-	cement
C_2S	-	Dicalcium silicate
C_2SH_x, $C_3S_2H_x$	-	hydrated calcium silicates
C_3A	-	Tricalcium aluminate
C_3AH_x, C_4AH_x	-	hydrated calcium aluminates
C_3S	-	Tricalcium silicate
C_4AF	-	Tetracalcium alumino-ferrite
$Ca(OH)_2$	-	hydrated lime
Ca^{2+}	-	Calcium ion
CaO	-	Calcium Oxide
$CC_{xy}(t)$,	-	cross-correlation function
CO_2	-	Carbon dioxide
C-S-H	-	calcium silicate hydrate
c_u	-	undrained shear strength

c_v	-	coefficient of consolidation
DMM	-	Deep Mixing Method
DSM	-	deep soil mixing
e	-	void ratio
e.g.	-	for example
E_{50}	-	secant modulus at 50 % of peak stress
E_o	-	initial tangent modulus
E_p	-	peak stress
f	-	frequency of the transmitted signal
Fe_2O_3	-	Iron Oxide
G_s	-	Specific gravity
G_o	-	Maximum shear modulus
i.e.	-	in other words
K^+	-	Potassium ion
K_2O	-	Potassium Oxide
kHz	-	kilohertz
L	-	effective distance traveled by the shear wave through the soil specimen
l	-	distance traveled
m	-	meter
Mg^{2+}	-	Magnesium Oxide

MgO	-	Magnesium Oxide
mm	-	milimeter
m_v	-	coefficient of volume compressibility
Na_2O	-	Sodium Oxide
NCL	-	normal consolidation line
NDT	-	Nondestructive test
OPC	-	Ordinary Portland cement
P	-	compressive force
P_c	-	preconsolidation pressure
POC	-	palm oil clinker
q_u	-	unconfined compressive strength
R^2	-	coefficient of correlation
RCff	-	column free-free tests
R_d	-	near field effect
RECESS	-	Research Centre for Soft Soils
RH	-	Rice husk
RHA	-	rice husk ash
RL	-	regression line
SASW	-	Spectral analysis surface wave
SiO_2	-	Silica

SO_3	-	Sulphur Trioxide
SO_4^{2+}	-	Sulfate ion
t	-	tonne
t	-	travel time
TiO_2	-	Titanium Oxide
t_o	-	first time of arrival
t_{pp}	-	first peak to peak time
UCS	-	unconfined compressive strength test
UTHM	-	Universiti Tun Hussein Onn Malaysia
v	-	Poisson's ratio
v_s	-	shear wave velocity
w	-	moisture content
W_s	-	dry weight
W_w	-	wet weight
$X(T)$	-	degree of correlation of the received signal
XRF	-	X-ray Fluorescence
$Y(T)$	-	transmitted signal
µm	-	micro meter
ρ	-	density
σ_y'	-	yield stress

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CHAPTER II

LITERATURE REVIEW

2.1 Soft Soil

Generally, the term soft soils include soft clay soils, soils with large fractions of fine particles such as silts, clay soils which have high moisture content, peat foundations, and loose sand deposits near or under the water table (Kamon and Bergado, 1992).

From the soil map of the study area, Parit Raja, Batu Pahat shown in Figure 2.1, it can be seen that the site sits on low humic gley soils, which is poorly drained soils deposited over coastal plains and in the valleys and flood plains of the larger rivers, of very variable fertility (Soil Map of Malaya, 1962).

Soft soils pose high moisture content, low shear strength and exhibits high compressibility. Utilizing such materials as a foundation material is almost impossible without some means of improving the adverse properties. Although several methods, such as prefabricated vertical drains, geotextile reinforcing, cement and lime stabilization, have been successfully implemented to treat such soils, there always remains the motivation for further improvement of the methods, especially in terms of efficiency and economics (Amer and Indra, 2007).

2.2 Soft Soil Stabilization with Deep Mixing

The research and development of current Deep Mixing Method (DMM) began in the late 1960s using lime as a stabilizing agent. DMM was put into practice in Japan and Nordic countries in the middle of the 1970s, and then spread to China and South East Asia. More than two decades of practice have seen the equipment improved, stabilizing agents changed, and the applications diversified (CDIT, 2002).

According to EuroSoilStab (2002), deep stabilization is a method to stabilize soft soils by adding dry or wet binders in order to reduce settlements and/or to improve the stability. The soil can be stabilized either by forming columns of stabilized soil (column stabilization) or by stabilizing the upper soil layers of organic soils (mass stabilization).

Columns with a very high strength giving high strength elements can be made. Such columns are brittle with a very low strain at failure and acts as bearing elements. The columns can also be made semi-hard with an appropriate strength level and not as brittle as the high strength columns (Holm, 1999). This means that the semi-hard columns can interact with the unstabilized soil between the columns, as schematically shown in Figure 2.2.

2.2.1 The Mixing Process

The most important factor in the success for application of DMM is ensuring that the injection and mixing processes of the stabilizing agents with the soil are thorough and effective, so that homogeneous, well-mixed, stabilized columns of soil are produced. The effectiveness of the mixing process relies on the use of augers consisting of appropriate and adequate cutting and mixing sections (Al-Tabba et al., 1999).

According to Larsson et al. (2005), there are three steps in the mixing process for the dry mixing method including the penetration of the mixing device, the dispersion of binders into the soil and molecular diffusion as shown in Figure 2.3.

The first part of the mixing process is the rotation and the penetration of the mixing device through the column. At this part, the soil structure is broken up. The second stage is the dispersion of stabilizing agent into the soft soil. The stages in the dispersion process are as follows:

i. Incorporation of dry stabilizing agent particles:
The dry stabilizing agent is distributed in the soft soil by compressed air. The stabilizing agent is forced into the soil through a hole in the kelly rod just above the mixing device.

ii. Wetting:
The air in the powder, agglomerates or aggregates is replaced when the dry stabilizing agent is incorporated with the soil as water is drawn from the surrounding soil. The air can also be replaced mechanically by the compaction device.

iii. Breakdown of agglomerates and aggregates:
When the stabilizing agent is wetted, the remaining agglomerates and aggregates must be broken up, to avoid lumps. This can be achieved by shearing the soil or by direct impact.

After installation of the columns, the mixing process continues as molecular diffusion. Different types of stabilizing agents have different properties with respect to molecular reaction.

2.3 Stabilizing Agent

According to CDIT (2002), the stabilizing agents used in practice are, in the majority of cases, Portland cement and limes, but dozens of stabilizing agents are now available on the market. Some of these newly developed special stabilizing agents are designed for the improvement of clay soils with high water contents or organic soils, for which ordinary cement or lime is not very effective. Some other stabilizing agents are designed for cases where the rate of strength increase has to be controlled for the convenience of the construction. These stabilizing agents react slowly with soil and exhibit smaller strength in the short term, but result in sufficiently high strength in the long term in comparison with ordinary cement.

2.3.1 Ordinary Portland Cement

Cement stabilization has been widely used to improve the engineering properties of the clayey soils (e.g. Broms 1999, Feng et al. 2001, Lorenzo and Bergado 2006, Xiao and Lee 2008).

Cement is a hydraulic type stabilizing agent. According to Bergado et al. (1996), there are two major chemical reactions which are induced by the addition of cement to clay and govern the soil cement stabilization process: the primary hydration reaction of the cement and water, and the secondary pozzolanic reactions between the limes released by the cement and the clay minerals. The primary hydration products are hydrated calcium silicates (C_2SH_x, $C_3S_2H_x$), hydrated calcium aluminates (C_3AH_x, C_4AH_x) and hydrated lime $Ca(OH)_2$. The first two of the hydration products listed above are the main cementitious products formed and the hydrated lime is deposited as a separate crystalline solid phase. These cement particles bind the adjacent cement grains together during hardening and form a hardened skeleton matrix, which encloses unaltered soil particles.

The soil silica and alumina, which are inherently acidic, will then be dissolved by the strong bases of cement compounds from the clay minerals and amorphous materials on the surface of the clay particle, which is similar to the reaction between a weak acid and a strong base. The hydrous silica and alumina will then gradually react with the calcium ions liberated from the hydrolysis of cement to form anew insoluble compound (secondary cementing product), which hardens when cured and thereby stabilize the soil. The transformation of Portland cement with addition of water into cement paste can be chemically illustrated as follows:

$$2(3CaO.SiO_2) + 6H_2O = 3CaO.2SiO_2.3H_2O + 3Ca(OH)_2 \tag{2.1}$$

(Tricalcium Silicate) (Water) (Tobermorite gel) (Calcium hydroxide)

$$2(2CaO.SiO_2) + 4H_2O = 3CaO.2SiO_2.3H_2O + Ca(OH)_2 \tag{2.2}$$

(Bicalcium Silicate) (Water) (Tobermorite gel) (Calcium hydroxide)

$$4CaO.Al_2O_3.Fe_2O_3 + 10H_2O + 2Ca(OH)_2 = 6CaO.Al_2O_3.Fe_2O_3.12H_2O \tag{2.3}$$

(Tetracalcium Aluminoferrite) (Water) (Calcium Hydroxide) (Calcium Aluminoferrite Hydrate)

$$3CaO.Al_2O_3 + 12H_2O + Ca(OH)_2 = 3CaO.Al_2O_3.Ca(OH)_2.12H_2O \tag{2.4}$$

(Tricalcium Aluminate) (Water) (Calcium Hydroxide) (Tetracalcium Aluminate Hydrate)

$$3CaO.Al2O3 + 10H2O + CaSO4.2H2O = 3CaO.Al_2O_3.CaSO4.12H_2O \tag{2.5}$$

(Tricalcium Aluminate) (Water) (Gypsum) (Calcium Monosulfoaluminate)

Bergado et al. (1996) further explained that the reactions which take place in soil-cement stabilization can be represented in the equation below. The reactions given are for tricalcium silicate (C_3S) only, because they are the most important constituents of Portland cement:

$$C_3S + H_2O \rightarrow \underset{\text{(Primary Cementitious Products)}}{C_3S_2H_x} + Ca(OH)_2 \quad (2.6)$$

$$Ca(OH)_2 \rightarrow Ca^{++} + 2(OH)^- \quad (2.7)$$

$$Ca^{++} + 2(OH)^- + SiO_2 \text{ (soil silica)} \rightarrow \underset{\text{(Secondary Cementitious Products)}}{CSH} \quad (2.8)$$

$$Ca^{++} + 2(OH)^- + Al_2O_3 \text{ (soil alumina)} \rightarrow \underset{\text{(Secondary Cementitious Products)}}{CAH} \quad (2.9)$$

2.3.2 Rice Husk

According to FAO (2009), global rice paddy production in 2009 was approximately 668 million tones and Malaysia's contribution was 2.45 million tones. Rice husk (RH) is the outer covering of the rice grain and is obtained during the milling process.

Commercially, RH produced in Malaysia is ground with broken rice to be used as animal feed. A large part of this agricultural by-product is burnt as fuel during rice processing and the resulting ash is sold as fertilizer. Other applications of RH are as silicon carbide whiskers, and as aggregates and fillers for concrete and board production. However, most RH is disposed of openly or in landfills, risking potential pollution and contamination, leading to further environmental problems (Johnson and Yunus, 2009).

Mehta (1992) stated that about 20 % of a dried rice paddy is made up of the rice husks. The current world production of rice paddy is around 500 million tons and hence 100 million rice husk are produced, as shown in Table 2.1. According to Lee et al. (2007), the amount of rice husk generated annually by the local rice mills in Malaysia is estimated to be 3.41 million cubic meter.

2.3.2.1 Properties and Pozzolanic Effect of Rice Husk

Pozzolanic materials are natural materials or by-products of industry, when mixed with water and alkali or alkaline earth hydroxide activators, react to form cementitious compounds (Marian and Raymond, 1999). Dwivedi et al. (2006) stated that when certain pozzolanic materials containing amorphous silica are added during the hydration of Portland cement, the reaction with lime produces additional amount of calcium silicate hydrate (C-S-H), the main cementing component.

Hypothetically, the high silica content of rice husks could contribute to the pozzolanic reaction of cement during hydration and enhance the bonding strength of the soil mass. In their work on mortar admixed with rice husks and cement, Jauberthie et al. (2000) observed that the pozzolanic role of rice husk was due to the existence of silica in amorphous and fine grained quartz in the mixture (Tables 2.2 to 2.4). Pozzolanic effect exhibits cementitious properties that increase the rate at which the material gains strength.

According to Sing et al. (2007), the chemical reaction between silica from rice husk with cement and soil is represented by Equations 2.10 and 2.11. Like a reaction between a week acid and a strong base, the strong base eventually dissolves the silica within the rice husk and clay soil to form hydrous silica. The hydrous silica will gradually react with the calcium ions produced from the cement hydrolysis to generate insoluble and cementitous materials known as secondary silicate. That means the solution of silica actually generates additional bonding forces in the cemented soil. Upon curing, the secondary cementitous products will harden and provide secondary bondage strength to the stabilized soil. This reaction is referred to as secondary pozzolanic reaction.

$$Ca(OH)_2 \longrightarrow Ca^{+} + 2(OH)^{-} \quad (2.10)$$

$$\underset{}{Ca^{+}} + 2(OH)^{-} + \underset{\text{Silica from rice husk and clay soil}}{SiO_2} \longrightarrow \underset{\text{Secondary calcium silicate hydrate}}{CaO.SiO_2.H_2O} \quad (2.11)$$

2.4 Measurement of Stiffness using Seismic Method

There are two categories of seismic wave: body waves, comprising compression (P) and shear (S) waves, and surface waves, which include Rayleigh (R) waves (Ortigao, 2007).

P-wave (often referred to as 'Pressure' wave) is a longitudinal wave in which the direction of motion of the particles is in the direction of propagation as shown in Figure 2.4 (a). On the other hand, S-wave (also referred to as 'Shear' wave), as shown in Figure 2.4 (b), is a transverse wave in which the direction of motion of the particles is perpendicular to the direction of propagation where the shear distortion is applied (GDS, 2005).

It should be noted that in saturated uncemented soils, the propagation of P-wave will represent a short term undrained loading. In such case the compressibility of the pore water will tend to dominate the compressibility behaviour of the soil. The result is that the measured P-wave velocity is likely to be close to that of water (i.e. 1500 m/s) since most of the energy will travel through the pore water and will not reflect the true undrained stiffness of the soil. Thus for stiffness measurements in soils, only shear wave velocities should be used since these are not affected by the compressibility of the pore fluid (Ortigao, 2007).

Geotechnical engineers generally require stiffness measurements to be made at different depths in order to determine a stiffness-depth profile. Ortigao (2007) had summarized the seismic methods employed in determination of stiffness-depth profiles, as shown in Table 2.5.

2.5 Measurement of v_s for Stabilized Soil

Measurements of v_s in various soils have attracted increased attention over the last decades (e.g Heineck et al. 2005, Leong et al. 2005, Karl et al. 2006 and Zhou and Chen 2007). However, comparatively few investigations on stabilized soils have been reported (Åhnberg and Holmén, 2009).

Hoyos et al. (2004) carried out a series of resonant column tests on chemically stabilized specimens of a sulfate-rich expansive clay and tested for the effects of different stabilizer types, stabilizer dosages, compaction moisture contents and confining pressures. Three stabilizers were experimented with: sulfate resistant type V cement, low calcium class F fly ash and lime mixed with polypropylene fibers.

Figure 2.5 shows the influence of compaction moisture content (w) on the dynamic response of stabilized specimens. The maximum shear modulus (G_o) from all stabilized specimens were highly susceptible to even slight changes in w, with the most dramatic cases observed in the cement-stabilized specimens. This can be attributed to the fact that stiffness enhancement in stabilized soils being highly dependent on the cementitious nature of each stabilizer. It can be noted that highly cementitious stabilizers, such as type V cement, yielded the highest G_o at higher levels of hydration. On the other hand, fly ash-based stabilization was not found to make significant contribution to G_o. Similar observation was made of the 5 % cement-and lime/fibers-based treatment. This shows that foreign elements with negligible or zero cementitious properties will not react chemically with the soil, hence resulting in marginal stiffness increase by their addition to soils.

Madhyannapu et al. (2010) reviewed the process and results of a quality management program performed during and immediately after the construction of two deep soil mixing (DSM) test sections. The SASW test method was performed to evaluate the degree of improvement achieved through the measurement of shear wave velocities of the columns and surrounding soils. From the test, the S-wave velocities recorded were 1.3 and 1.5 times of those recorded in untreated site, evidence of considerable improvement in stiffness in and around the DSM columns compared to original soil site.

2.6 Bender Element (BE) Test

Bender elements are piezoelectric ceramic devices used in geotechnical laboratory dynamic testing. The use of bender elements to measure shear wave velocity in soils was first attempted by Shirley and Hampton (1977). Bender elements work as cantilevers which flex or bend when excited by the voltage differential of an electric signal and vice versa, i.e., they bend when excited by a voltage input, and generate a voltage differential when forced to bend. The transducers are used in pairs, where one operates as the transmitter and the other the receiver.

The bender elements work by having the transmitter transform an input electric signal into a mechanical motion, which in turn disturbs the medium in which it is embedded. This disturbance propagates through the medium in the form of mechanical waves and is detected by the receiver. The receiver is then forced to bend, and consequently generating an output electric signal due to its piezoelectric properties mentioned above. The input voltage or transmitted signal, and the received signal are both recorded on a timescale, allowing the travel time of the shear waves to be determined.

The primary objective of the BE test is to measure the travel time of a wave propagating through a soil specimen from the transmitting to the receiving transducers (Shirley and Hampton, 1977). The shear wave velocity, v_s, can be determined by

$$v_s = L/t \tag{2.12}$$

where L is the effective distance traveled by the shear wave through the soil specimen, which equals the tip to tip distance between the bender elements and t is the travel time of the shear wave. Accordingly, G_o can be calculated from the shear wave velocity as follows

$$G_o = \rho v_s^2 \tag{2.13}$$

where ρ is the density of soil.

2.6.1 BE Test in Geotechnical Testing

According to Puppala et al. (2006) and Chan (2009), one of the main advantages of the shear wave velocity measurement is that the numbers of specimens required are significantly reduced, because it provides a repeatable measurement of the same soil specimen under varying conditions, such as curing period.

Due to this advantage, BE test has been much adopted for measuring shear wave velocity, such as demonstrated by Sahaphol and Miura (2005), Wang et al. (2007), Landon et al. (2007) and Zhou et al. (2008).

Sahaphol and Miura (2005) reported that the maximum shear modulus (G_o) of soils is one of the important parameters in small strain level geotechnical problems (i.e. the study of earthquake effects and soil structure interaction). In their study of crushable volcanic soils, measurements of G_o from bender element and cyclic triaxial tests were compared. The G_o values evaluated with the bender elements are plotted against those from the cyclic triaxial tests in Figure 2.6. Both measurements agreed well, though stiffness in the cyclic test was derived from direct measurement of the stress-strain change while the bender element test used transient waves.

As the BE test generally requires that the transducer pair be plugged on to opposite ends of a test specimen, there is a valid concern of the effect of such intrusion on the quality of the measurements made. Chan et al. (2010) conducted a study in this direction by varying the receiver BE's penetration depth and rate, confining pressure, as well as the test specimen size, but with a fixed aspect ratio of 2. Based on the data collected (Figure 2.7), it was concluded that BE installation causes almost no disturbance irrespective of the factors mentioned above, hence can be readily performed on clay samples in the laboratory and field with little sensitivity to such variations.

2.6.2 Determination of Shear Wave Travel Time

The travel time of the shear wave between two points in a soil specimen can be used to calculated shear wave velocity and hence shear stiffness. However, the proposed methods that use the time-domain estimates to determine the travel time are controversial.

The detection of travel time is not straightforward and this has resulted in different researchers using a range of methods to obtain the correct travel time. For example, Jovičić et al. (1996) has recommended using the first time of arrival method (Section 2.6.3). However, they stress that great care is required to reduce system noise, and the pulse frequency must be adjusted to minimize near field affects (Section 2.6.4).

Jovičić et al. (1996) also suggested to use a sine pulse instead of a square one as the input signal when performing the BE test. Figure 2.8 shows an example of the received trace when square wave is used. As a square wave is composed of a spectrum of different frequencies, this result in significant difficulty of identifying the characteristic points on the received signal, like the point of first deflection (0) and the reversal point (1).

Chan (2005) grouped the techniques to ascertain the travel time into two main categories: time domain and frequency domain. The time domain methods refer to plots of electrical signals versus time, while the frequency domain methods involve analyzing the spectral breakdown of the signals and comparing phase shifts of the components. Chan (2005) also explored the different techniques from both groups in stabilized clay specimens and drew the conclusion that neither of the methods gives more consistent results.

Kumar and Madhusudhan (2010) performed BE tests on dry sand samples at different relative densities and effective confining pressures. Three methods of interpretations, namely, (1) the first time of arrival, (2) the first peak to peak, and (3) the cross-correlation method, were employed. The explanation of these methods can be referred to in Section 2.6.3.

A comparison was made by the authors among the travel times obtained by the different methods adopted for analyzing the results of the BE testing. The travel time (t) was also obtained by using the resonant column method. Figure 2.9 shows the comparison of the travel times from the different methods associated with three different values of effective consolidation pressures. From the results, the first peak to peak, the cross-correlation method and the resonant column tests provide results close to one another. However, the first time of arrival method sometimes gave a much different result compared to the other methods. This indirectly suggests the subjectivity and inconsistency of the method for determining the shear wave arrival time, due very much in part to the difficulty of selecting the ‘correct’ point of departure.

2.6.3 Methods for Determining Travel Time

A brief description of three most common methods for determining travel time is provided as follows:

i. First time of arrival

   The first time of arrival (t_o) refers to the time between the start of the transmitted signal and the start of the major cycle of the received signal (Viggiani and Atkinson 1995, Jovičić et al.1996, Arulnathan et al. 1998, Kumar and Madhusudhan 2010). Figure 2.10 (a) shows an example of this method.

ii. First peak to peak time

   The first peak to peak time (t_{pp}) method (Viggiani and Atkinson 1995, Arulnathan et al. 1998, Kumar and Madhusudhan 2010) is based on to the time between the peak of the transmitted signal and the first peak of the cycle of the received signal (Figure 2.10a).

iii. Cross-correlation

   The cross-correlation method essentially refers to the cross-correlation function $CC_{xy}(t)$, which is a measure of the degree of correlation of the received signal, $X(T)$, and the transmitted signal, $Y(T)$, versus the time shift (τ). An example of the cross-correlation function is shown in Figure 2.10 (b). Full details for determination of this method can be referred to in Viggiani and Atkinson (1995).

2.6.4 Near Field Effect

The shape of the received signal is affected by the presence of near field effect, which can distort the received signal and makes identification of the travel time difficult, particularly in the time domain (Jovičić et al.1996 and Arulnathan et al. 1998). Sanches-Salinero et al. (1986) recognized that the R_d ratio controls the shape of the received signal through the degree of attenuation that occurs as the wave travels through the sample. R_d is given by:

$$R_d = l/\lambda = l\,f/v_s \tag{2.14}$$

Where l is the distance traveled and f is the frequency of the transmitted signal. Figure 2.11 shows an initial downward deflection of a trace, corresponding to low values of R_d, before the actual arrival of the shear wave. This is a manifestation of the near field effect, which is almost absent in higher range of R_d values.

2.6.5 Bender Element (BE) Test in Stabilized Soils

Research on stabilized soils with bender elements has gained increased attention in recent years (e.g Heineck et al. 2005, Puppala et al. 2006, Chan 2007, Hird and Chan 2008, Åhnberg and Holmén 2009 and Asaka and Abe 2009).

Bender element tests were conducted by Puppala et al. (2006) on cement and lime stabilized clay specimens and the results were analyzed to study the variation in stiffness properties of clay specimens at different chemical stabilizers. The authors reported that the G_o of untreated clay is low and in the range of 20 – 50 Mpa. The authors also reported that cement stabilized clay provided rapid enhancement of G_o with respect to curing time, whereas lime stabilized clay did not exhibit an immediate increase in G_o. These indicate that the time required for pozzolanic reactions was longer for lime stabilization than for the cement stabilization.

The change in shear wave signals throughout the 7-day curing period on stabilized clays was studied by Chan (2007) using BEs. The results showed that apart from the expected shorter travel time, the amplitude of the signal improved over time too. These two observations combined reflect the increased stiffness of the material as hydration of the cement proceeded with time.

The quality of cement-stabilized grounds was inspected at two construction sites by Asaka and Abe (2009), where the BE test procedure is summarized in Figure 2.12. The stabilizers used for Sites A and B were blast-furnace slag and low Cr (VI) leaching respectively. BE test results showed significant v_s increment in Site A compared to Site B, indicating the influence of difference stabilizer type on the same soil type.

2.7 Unconfined Compressive Strength (UCS)

The test procedure for UCS test was as described in BS 1377: 1990: Part 7, where a cylindrical specimen of soil is subjected to a steadily increasing axial load until failure occurs. The axial load is the only force or stress which is applied (Figure 2.13). Equation for q_u is as stated in Eq. 2.15. Table 2.6 shows the description of clays according to the undrained shear strength (c_u) values where c_u = q_u/2 (Nagaraj and Miura, 2001).

$$q_u = P/A \tag{2.15}$$

Where;

q_u = unconfined compressive strength

P = compressive force

A = cross section area

2.7.1 UCS Test on Stabilized Soil

The unconfined compressive strength (q_u) is most frequently used as an index of the strength of stabilized soil (Lee and Lee, 2002). In spite of the different stabilizers used, many researchers have adopted the UCS test for its simplicity. Attom (2008) studied the strength characteristics of Irbid clayey soil mixed with iron filling and iron filling-cement mixture. UCS test results showed increased q_u and secantial modulus of elasticity with the addition of both stabilizers, but the iron filling-cement mixture was found to be more effective between the two.

Tang et al. (2007) carried out a study on strength of short polypropylene fiber reinforced and cement stabilized clayey soil. Figure 2.14 shows the effect of fiber content on the uncemented soil and cemented soil after curing for 28 days. It is indicated that fiber plays a more important role in cemented soil than it does in uncemented soil where the influenced of fiber inclusion on uncemented soils is not significant. This is because for fiber-reinforced cemented soil, the interactions between the fiber surface and the hydrated products make main contribution to the higher strength than the uncemented soil.

Tan et al. (2002) stated that a primary concern in the characterization study of improved soil is to estimate the expected increase in improvement, usually based on properties after 1 day or 7 days of curing. This will give an early indication of whether a particular mix proportion can achieve the design strength at 28 days, the usual benchmark for determination strength. Figure 2.15 shows the relationship between normalized strength at 28 days with the normalized strengths at other time intervals. The relationships are generally linear and range between 1.2 to 2.9 times of strength for 7 - 28 days comparison.

Kitazume and Nishimura (2009) presented the variations of q_u with temperature observed for the Kawasaki clay as shown in Figure 2.16 for different binder contents and curing days. The results indicate that the higher curing temperature and longer curing period led to higher strength achieved.

2.8 Correlation between Shear Wave Velocity and UCS Test

Madhyannapu et al. (2010) developed an empirical correlation between the UCS (q_u) versus shear wave velocity (v_s) from the laboratory test conducted on lime-cement stabilized expansive soils. The main intent of this correlation was to interpret the strength properties from v_s and vice versa. The correlation of q_u versus v_s is presented in Figure 2.17 (Site 1 and Site 2). The authors claimed that the correlations developed are useful in field quality assessments where the strength is deduced from the v_s-q_u correlation. It was further shown that the predicted q_u corresponded closely to the measured strengths of UCS tests (Figure 2.18).

The usefulness of the q_u-v_s relationship was also demonstrated by Åhnberg and Holmén (2009), with stabilized soil samples collected from more than twenty different geotechnical engineering projects. The types of soil in their research range from very soft to sand. The stabilizing agents used included different types of cement, lime, ground granulated blast-furnace slag and fly ash. Figure 2.19 shows the variation in shear wave velocity, evaluated from bender element and the resonant column free-free tests (RCff), with the strengths from UCS tests. The results shown in Figure 2.24 relate only to specimens having water contents greater than 40 %. The authors also stated that the relationship between q_u and v_s in their research agree fairly well with those reported by other researchers as showed in Figure 2.20.

Clearly, such correlations are time-saving and marginally labour-intensive for quality control of stabilization on site. However, it should be cautioned that different soil and binder types result in different correlations, making site specific correlations necessary for reliable predictions to be made from the v_s measured.

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